

Module 1

GEOTECHNICAL PROPERTIES OF SOIL AND OF REINFORCED SOIL (Lectures 1 to 4)

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Lectures 1

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1.1 INTRODUCTION

The design of foundations of structures such as buildings, bridges, and dams generally requires a knowledge of such factors as (a) the load that will be transmitted by the superstructure to the foundation system, (b) the requirements of the local buildings code, (c) the behavior and stress-related deformability of soils that will support the foundation system, and d the geological conditions of the under consideration. To a foundation engineer, the last two factors are extremely important because they concern soil mechanics.

The geotechnical properties of a soil-such as the grain-size distribution, plasticity, compressibility, and shear strength-can are assessed by proper laboratory testing. And, recently, emphasis has been placed on in situ determination of strength and deformation properties of soil, because this process avoids the sample disturbances that occur during field exploration. However, under certain circumstances, all of the needed parameters cannot be determined or are not determined because of economic or other reasons. In such cases, the engineer must make certain assumptions regarding

the properties of the soil. The assessment of the accuracy of soil parameters-whether they were determined in the laboratory and the field or were assumed-the engineer must have a good grasp of the basic principles of soil mechanics. At the same time, he or she must realize that the natural soil deposits on which foundation are constructed are not homogeneous in most cases. Thus the engineer must have a thorough understanding of the geology of the area-that is, the origin and nature of soil stratification and also the groundwater conditions. Foundation engineering is a clever combination of soil mechanics, engineering geology, and proper judgment derived from past experience. To a certain extent, it may be called an “art”.

When determining which foundation is the most economical, the engineer must consider the superstructure load, the subsoil conditions, and the desired tolerable settlement. In general, foundations of buildings and bridges may be divided into two major categories: (1) shallow foundations and (2) deep foundations. Spread footings, wall footings, and mat foundations are all shallow foundations. In most shallow foundations, the depth of embedment can be equal to or less than three to four the width of the foundation. Pile and drilled shaft foundations are deep foundations.

This chapter serves primarily as a review of the basic geotechnical properties of soil. It includes topics such as grain-size distribution, plasticity, and soil classification effective stress, consolidation, and shear strength parameters. It is based on assumption that you have already been exposed to these concepts in a basic mechanics course.

1.2 GRAIN-SIZE DISTRIBUTION

In any soil mass, the sizes of various soil grains vary greatly. To classify a properly, you must know its grain-size distribution. The grain-size distribution of coarse-grained soil is generally determined by means of sieve analysis. For a fine grained soil, the grain-size distribution can be obtained by means of hydrometer analysis. The fundamental features of these analyses are presented in this section. For detailed descriptions, see any soil mechanics laboratory manual (for exam, Das, 1997).

1.2.1 Sieve Analysis

A sieve analysis is conducted by taking a measured amount of dry, well-pulverized soil and passing it through a stack of progressively finer sieves with a pan at bottom. The amount of soil retained on each sieve is measured, and the cumulative percentage of soil passing through each sieve is determined. This percentage generally referred to as percent finer. Table 1 contains a list of U. S. sieve number and the corresponding size of their hole openings. These sieves are commonly used for the analysis of soil classification purposes.

The percent finer for each sieve determined by a sieve analysis is plotted semi logarithmic graph paper, as shown in **figure 1.1**. Note that the grain diameter D , is plotted on the logarithmic scale, and the percent finer is plotted on the arithmetic scale.

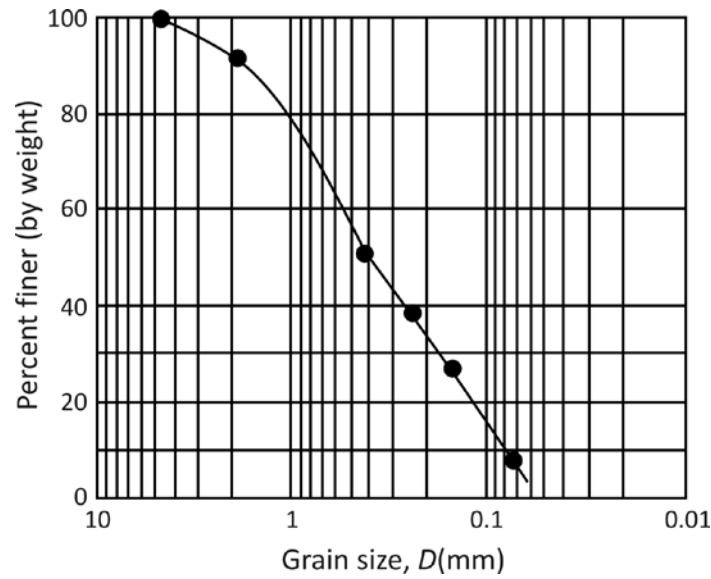


Figure 1.1 Grain-size distribution curve of a coarse grained soil obtained from sieve analysis

Two parameters can be determined from the grain-size distribution curves coarse-grained soils: (1) the uniformity coefficient (C_u) and (2) the coefficient gradation, or coefficient of curvature (C_z). These coefficients are

Table U. S. Standard Sieve Sizes

Sieve no.	Opening (mm)
4	4.750
6	3.350
8	2.360
10	2.000
16	1.180
20	0.850
30	0.600
40	0.425
50	0.300
60	0.250
80	0.180
100	0.150
140	0.106
170	0.088
200	0.075
270	0.053

$$C_u = \frac{D_{60}}{D_{10}} \quad [1.1]$$

$$C_z = \frac{D_{30}^2}{D_{60} \times D_{10}} \quad [1.2]$$

where D_{10} , D_{30} , and D_{60} are the diameters corresponding to percents fines the 10, 30, and 60%, respectively.

For the grain-size distribution curve shown in **figure 1.1**, $D_{10} = 0.08$ mm, $D_{30} = 0.17$ mm, and $D_{60} = 0.57$ mm. Thus the values of C_u and C_z are

$$C_u = \frac{0.57}{0.08} = 7.13$$

$$C_z = \frac{0.17^2}{(0.57)(0.08)} = 0.63$$

Parameters C_u and C_z are used in the Unified Soil Classification System, which is described later in this chapter.

Hydrometer Analysis

Hydrometer analysis is based on the principle of sedimentation of soil particles in water. This test involves the use of 50 grams of dry, pulverized soil. A deflocculating agent is always added to the soil. The most common deflocculating agent used with the hydrometer is 125 cc of 4% solution of sodium hexametaphosphate. The soil is allowed to soak for at least 16 hours in the deflocculating agent. After the soaking period, distilled water is added, and the soil-deflocculating agent mixture is thoroughly agitated. The sample is then transferred to a 1000-ml glass cylinder. More distilled water is added to the cylinder to fill it to the 1000-ml mark, and the mixture is again thoroughly agitated. A hydrometer is placed in the cylinder to measure—usually over a 24-hour period—the specific gravity of the soil-water suspension in the vicinity of its bulb (**figure 1.2**). Hydrometers are calibrated to show the amount of soil that is still in suspension at any given time, t . The large diameter of the soil particles still in suspension at time t can be determined using Stoke's law:

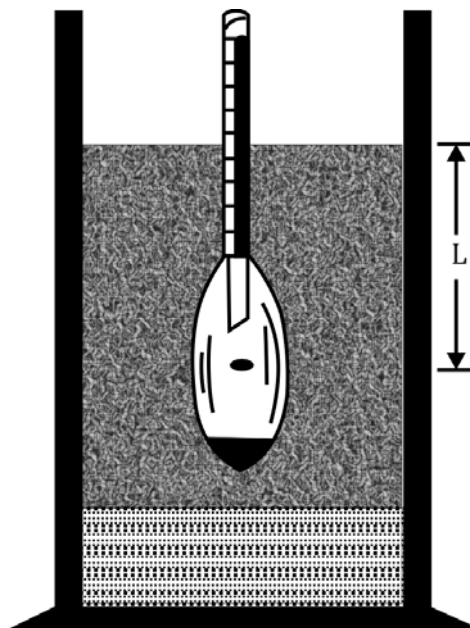


Figure 1.2 Hydrometer analysis

$$D = \sqrt{\frac{18\eta}{(G_s-1)\gamma_w}} \sqrt{\frac{L}{t}} \quad [1.3]$$

Where

D = diameter of the soil particle

G_s = specific gravity of soil solids

η = viscosity of water

γ_w = unit weight of water

L = effective length (that is, length measured from the water surface in the cylinder to the center of gravity of the hydrometer, see figure 1.2)

t = time

Soil particles having diameters larger than those calculated by equation (3) would have settled beyond the zone of measurement. In this manner, with hydrometer readings taken at various times, the soil percent finer than a given diameter D can be calculated, and a grain-size distribution plot can be prepared. The sieve and hydrometer techniques may be combined for a soil having both coarse-grained and fine-grained soil constituents.

SIZE LIMITS FOR SOILS

Several organizations have attempted to develop the size limits for gravel, sand, silt, and clay based on the grain sizes present in soils. Table 2 presents the size limits recommended by the American Association of State Highway and Transportation Officials (AASHTO) and the United Soil Classification systems (Corps of Engineers, Department of the Army, and Bureau of Reclamation): table 2 shows that soil particles smaller than 0.002 mm have been classified as clay. However, clays by nature are cohesive and can be rolled into a thread when moist. This property is caused by the presence of clay minerals such as kaolinite, illite, and montmorillon: In contrast, some minerals such as quartz and feldspar may be present in a soil particle sizes as small as clay minerals. But these particles will not have the cohesive property of clay minerals. Hence they are called clay-size particles, not clay particles.

Table 2 Soil Separate Size Limits

Classification system	Grain size (mm)
Unified	Gravel: 75 mm to 4.75 mm Sand: 4.75 mm to 0.075 mm Silt and clay (fines): < 0.075 mm
AASHTO	Gravel: 75 mm to 2 mm Sand: 2 mm to 0.05 mm Silt: 0.05 mm to 0.002 mm Clay: < 0.002 mm

WEIGHT-VOLUME RELATIONSHIPS

In nature, soils are three-phase systems consisting of solid soil particles, water, an air (or gas). To develop the weight-volume relationships for a soil, the three phases can be separated as shown in **figure 1.3**. Based on this separation, the volume relationships can be defined in the following manner.

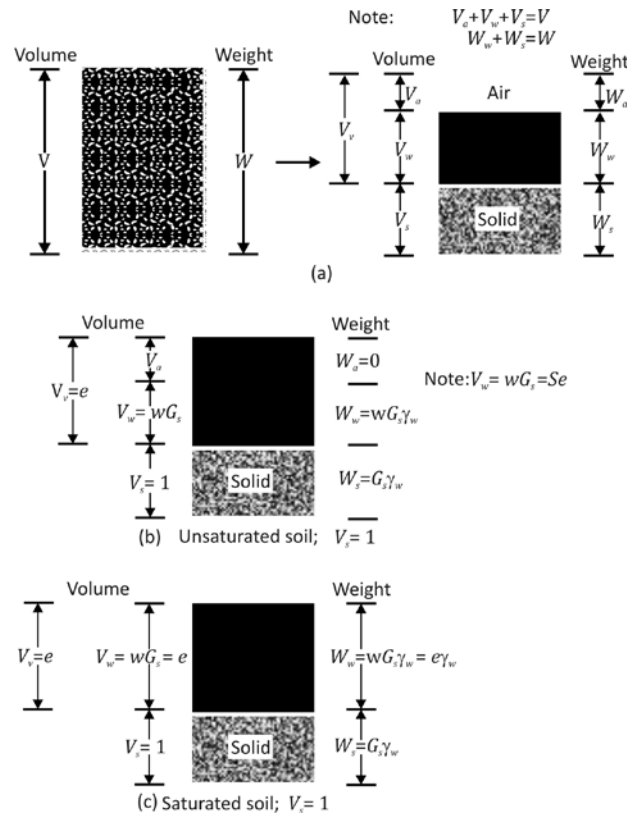


Figure 1.3 Weight-volume relationships

Void ratio, e , is the ratio of the volume of voids to the volume of soil solids a given soil mass, or

$$e = \frac{V_v}{V_s} \quad [1.4]$$

Where

V_v = volume of voids

V_s = volume of soil solids

Porosity, n , is the ratio of the volume of voids to the volume of the soil specimen,

$$n = \frac{V_v}{V} \quad [1.5]$$

Where V = total volume of soil

Moreover,

$$n = \frac{V_v}{V} = \frac{V_v}{V_s + V_v} = \frac{\frac{V_v}{V_s}}{\frac{V_s + V_v}{V_s}} = \frac{e}{1+e} \quad [1.6]$$

Degree of saturation, S , is the ratio of the volume of water in the void spaces the volume of voids, generally expressed as a percentage, or

$$S(\%) = \frac{V_w}{V_v} \times 100 \quad [1.7]$$

Where

$$V_w = \text{volume of water}$$

Note that, for saturated soils, the degree of saturation is 100%.

The weight relationships are moisture content, moist unit weight, dry unit weight, and saturated unit weight. They can be defined as follows:

$$\text{Moisture content} = w(\%) = \frac{W_w}{W_s} \times 100$$

Where

$$W_s = \text{weight of soil solids} \quad [1.8]$$

$$W_w = \text{weight of water}$$

$$\text{Moist unit weight} = \gamma = \frac{W}{V} \quad [1.9]$$

Where

$$W = \text{total weight of the soil specimen} = W_s + W_w$$

The weight of air, W_a , in the soil mass is assumed to be negligible.

$$\text{Dry unit weight} = \gamma_d = \frac{W_s}{V} \quad [1.10]$$

When a soil mass is completely saturated (that is, all void volume is occur by water), the moist unit weight of a soil [equation (9)] becomes equal to the saturated unit weight (γ_{sat}). so $\gamma = \gamma_{\text{sat}}$ if $V_v = V_w$.

More useful relations can now be developed by considering a representative soil specimen in which the volume of soil solids is equal to unity, as shown in **figure 1.3b**. Note that if $V_s = 1$, from equation (4), $V_v = e$ and the weight of the soil solids:

$$W_w = G_s \gamma_w$$

Where

$G_s =$ specific gravity of soil solids

$\gamma_w =$ unit weight of water (9.81 kN/m³, or 62.4 lb/ft³)

Also, from equation (8), the weight of water $W_w = wW_s$. Thus, for the soil specimen under consideration, $W_w = wW_s = wG_s \gamma_w$. Now, for the general relation for moist unit weight given in equation (9),

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_v} = \frac{G_s \gamma_w (1+w)}{1+e} \quad [1.11]$$

Similarly, the dry unit weight [equation (10)] is

$$\gamma_d = \frac{W_s}{V} = \frac{W_s}{V_s + V_v} = \frac{G_s \gamma_w}{1+e} \quad [1.12]$$

From equations (11) and (12), note that

$$\gamma_d = \frac{\gamma}{1+w} \quad [1.13]$$

If a soil specimen is completely saturated as shown in **figure 1.3c**,

$$V_v = e$$

Also, for this case

$$V_v = \frac{W_w}{\gamma_w} = \frac{wG_s \gamma_w}{\gamma_w} = wG_s$$

Thus

$$e = wG_s \text{ (for saturated soil only)} \quad [1.14]$$

The saturated unit weight of soil becomes

$$\gamma_{\text{sat}} = \frac{W_s + W_w}{V_s + V_v} = \frac{G_s \gamma_w + e \gamma_w}{1+e} \quad [1.15]$$

Relationships similar to equations (11, 12 and 15) in terms of porosity can also be obtained by considering a representative soil specimen with a unit volume.

These relationships are

$$\gamma = G_s \gamma_w (1 - n)(1 + w) \quad [1.16]$$

$$\gamma_d = (1 - n)G_s \gamma_w \quad [1.17]$$

$$\gamma_{sat} = [(1 - n)G_s + n]\gamma_w \quad [1.18]$$

Table 3 provides several useful relationships for γ , γ_d , and γ_{sat} .

Except for peat and highly organic soils, the general range of the values of specific gravity of soil solids (G_s) found in the nature is rather small. Table 4 gives some representative values. For practical purposes, a reasonable value can be assumed in lieu of running a test.

Table 5 presents some representative values for the void ratio, dry unit weight, and moisture content (in a saturated state) of some naturally occurring soils. Note that in most cohesionless soils the void ratio varies from about 0.4 to 0.8. The dry unit weight in these soils generally fall within a range of about 90 – 120 lb/ft³ (14 – 19 kN/m³).

RELATIVE DENSITY

In granular soils, the degree of compaction in the field can be measured according to relative density, D_r , which is defined as

$$D_r(\%) = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100 \quad [1.19]$$

Where

e_{max} = void ratio of the soil in the loosest state

e_{min} = void ratio in the densest state

e = in situ void ratio

Table 3 Various Forms of Relationships for γ , γ_d , and γ_w

Given	Moist unit weight, γ Relationship
w, G_s, e	$\frac{(1 + w)G_s \gamma_w}{1 + e}$
S, G_s, e	$\frac{(G_s + Se)\gamma_w}{1 + e}$
w, G_s, S	$\frac{(1 + w)G_s \gamma_w}{1 + \frac{wG_s}{S}}$
w, G_s, n	$G_s \gamma_w (1 - n)(1 + w)$
S, G_s, n	

	$G_s \gamma_w (1 - n) + n S \gamma_w$
Dry unit weight (γ_d)	
γ, w	$\frac{\gamma}{1 + w}$
G_s, e	$\frac{G_s \gamma_w}{1 + e}$
G_s, n	$G_s \gamma_w (1 - n)$
G_s, w, S	$\frac{G_s \gamma_w}{1 + \left(\frac{w G_s}{S}\right)}$
e, w, S	$\frac{e S \gamma_w}{(1 + e) w}$
γ_{sat}, n	$\frac{\gamma_{sat} - n \gamma_w}{G_s - 1}$
γ_{sat}, G_s	$\frac{(\gamma_{sat} - \gamma_w) G_s}{G_s - 1}$
Saturated unit weight (γ_{sat})	
G_s, e	$\frac{(G_s + e) \gamma_w}{1 + e}$
G_s, n	$[(1 - n) G_s + n] \gamma_w$
G_s, W_{sat}	$\left(\frac{1 + w_{sat}}{1 + w_{sat} G_s}\right) G_s \gamma_w$
e, w_{sat}	$\left(\frac{e}{w_{sat}}\right) \left(\frac{1 + w_{sat}}{1 + e}\right) \gamma_w$
n, w_{sat}	$n \left(\frac{1 + w_{sat}}{w_{sat}}\right) \gamma_w$
γ_d, e	$\gamma_d + \left(\frac{e}{1 + e}\right) \gamma_w$
γ_d, n	$\gamma_d + n \gamma_w$
γ_d, S	$\left(1 - \frac{1}{G_s}\right) \gamma_d + \gamma_w$
γ_d, W_{sat}	$\gamma_d (1 + w_{sat})$

Table 4 Specific Gravities of Some Soils

Soil type	G_s
Quartz sand	2.64-2.66
Silt	2.67-2.73
Clay	2.70-2.9
Chalk	2.60-2.75
Loess	2.65-2.73
Peat	1.30-1.9

Table 5 Typical Void Ratio, Moisture Content, and Dry Unit Weight for Some Soils

Type of soil	Void ratio e	Natural moisture content saturated in	Dry unit weight, γ_d	
			(lb/ft ³)	(kN/m ³)

		condition (%)		
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	120	19
Stiff clay	0.6	21	108	17
Soft clay	0.9-1.4	30-50	73-92	11.5-14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5-3.2	90-120	38-51	6-8
Glacial till	0.3	10	134	21

The values of e_{max} are determined in the laboratory in accordance with the test procedures outlined in the American Society for Testing and Materials, ASTM Standards (1997), Test Designation D-4254).

The relative density can also be expressed in terms of dry unit weight, or

$$D_r(\%) = \left\{ \frac{\gamma_d - \gamma_{d(\min)}}{\gamma_{d(\max)} - \gamma_{d(\min)}} \right\} \frac{\gamma_{d(\max)}}{\gamma_d} \times 100 \quad [1.20]$$

Where

γ_d = in situ dry unit weight

$\gamma_{d(\max)}$ = dry unit weight in the densest state; that is, when the void ratio is e_{min}

$\gamma_{d(\min)}$ = dry unit weight in the loosest state; that is, when the void ratio is e_{max}

The denseness of a granular soil is sometimes related to its relative density. Table 6 gives a general correlation of the denseness and D_r . For naturally occurring sands, the magnitudes of e_{max} and e_{min} v[equation (9)] may vary widely. The main reasons for such wide variations are the uniformity coefficient, C_u , and the roundness of the particles, R . the uniformity coefficient is defined in equation (1). Roundness is defined as

Table 6 Denseness of a Granular Soil

Relative density, $D_r(\%)$	Description
0-20	Very loose
20-40	Loose
40-60	Medium
60-80	Dense
80-100	Very dense

$$R = \frac{\text{minimum radius of the particle edges}}{\text{inscribed radius of the entire particle}} \quad [1.21]$$

Measuring R is difficult, but it can be estimated. **Figure 1.4** shows the general range of the magnitude of R with particle roundness. **Figure 1.5** shows the variation of e_{\max} and e_{\min} with the uniformity coefficient for various values of particle roundness (Youd, 1973). This range is applicable to clean sand with normal to moderate skewed particle-size distribution.

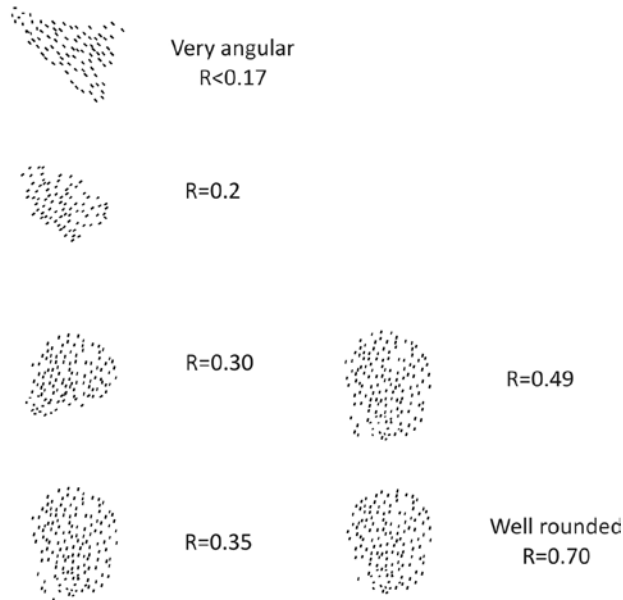


Figure 1.4 General range of the magnitude of R

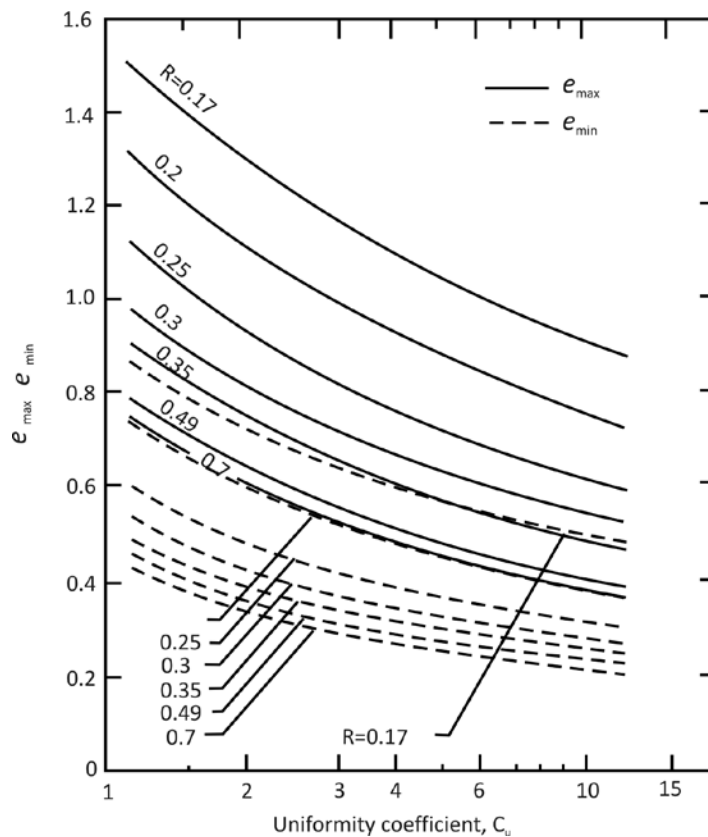


Figure 1.5 Approximate variation of e_{\max} and e_{\min} with uniformity coefficient (based on Youd, 1973)

Example 1

A 0.25 ft³ moist soil weights 30.8 lb. when dried in an oven, this soil weights 28.2 lb. given $G_s = 2.7$. Determine the

- Moist unit weight, γ
- Moisture content, w
- Dry unit weight, γ_d
- Void ratio, e
- Porosity, n
- Degree of saturation, S

Solution**Part a. Moist Unit Weight**

From equation (9),

$$\gamma = \frac{W}{V} = \frac{30.8}{0.25} = 123.2 \text{ lb/ft}^3$$

Part b. Moisture Content

$$w = \frac{W_w}{W_s} = \frac{30.8 - 28.2}{28.2} \times 100 = 9.2\%$$

Part c. Dry Unit Weight

From equation (10),

$$\gamma_d = \frac{W_s}{V} = \frac{28.2}{0.25} = 112.8 \text{ lb/ft}^3$$

Part d. Void Ratio

From equation (4),

$$e = \frac{V_v}{V_s}$$

$$V_s = \frac{W_s}{G_s \gamma_w} = \frac{28.2}{(2.67)(62.4)} = 0.169 \text{ ft}^3$$

$$e = \frac{0.25 - 0.169}{0.169} = 0.479$$

Part e. Porosity

From equation (6),

$$n = \frac{e}{1 + e} = \frac{0.479}{1 + 0.479} = 0.324$$

Part f. Degree of Saturation

From equation (7),

$$S(\%) = \frac{V_w}{V_v} \times 100$$

$$V_v = V - V_s = 0.25 - 0.169 = 0.081 \text{ ft}^3$$

$$V_w = \frac{W_w}{\gamma_w} = \frac{30.8 - 28.2}{62.4} = 0.042 \text{ ft}^3$$

$$S = \frac{0.042}{0.081} \times 100 = 51.9\%$$

Example 2

A soil has a void ratio of 0.72, moisture content = 12%, and $G_s = 2.72$. Determine

- Dry unit weight (kN/m^3)
- Moist unit weight (kN/m^3)
- Weight of water in kN/m^3 to be added to make the soil saturated

Solution**Part a. Dry Unit Weight**

From equation (12),

$$\gamma_d = \frac{G_s \gamma_w}{1 + e} = \frac{(2.72)(9.81)}{1 + 0.72} = 15.51 \text{ kN/m}^3$$

Part b. Moist Unit Weight

From equation (11),

$$\gamma = \frac{G_s \gamma_w (1 + w)}{1 + e} = \frac{(2.72)(9.81)(1 + 0.12)}{1 + 0.72} = 17.38 \text{ kN/m}^3$$

Part c. Mass of Water to be Added

From equation (15),

$$\gamma_{\text{sat}} = \frac{(G_s + e) \gamma_w}{1 + e} = \frac{(2.72 + 0.72)(9.81)}{1 + 0.72} = 19.62 \text{ kN/m}^3$$

$$\text{Water to be added} = \gamma_{\text{sat}} - \gamma = 19.62 - 17.38 = 2.24 \text{ kN/m}^3$$

Example 3

The maximum and minimum dry unit weights of a sand are 17.1 kN/m^3 and 14.2 kN/m^3 , respectively. The sand in the field has a relative density of 70% with a moisture content of 8%. Determine the moist unit weight of the sand in the field.

Solution

From equation (20),

$$D_r = \left[\frac{\gamma_d - \gamma_{d(\min)}}{\gamma_{d(\max)} - \gamma_{d(\min)}} \right] \left[\frac{\gamma_{d(\max)}}{\gamma_d} \right]$$

$$0.7 = \left[\frac{\gamma_d - 14.2}{17.1 - 14.2} \right] \left[\frac{17.1}{\gamma_d} \right]$$

$$\gamma = \gamma_d(1 + w) = 16.11 \left(1 + \frac{8}{100} \right) = 17.4 \text{ kN/m}^3$$

ATTERBERG LIMITS

When a clayey soil is mixed with an excessive amount of water, it may flow like a semiliquid. If the soil is gradually dried, it will behave like a plastic, semisolid, or solid material depending on its moisture content. The moisture content, in percent, at which the soil changes from a liquid to a plastic state, is defined as the liquid limit (LL). Similarly, the moisture contents, in percent, at which the soil change from a plastic to a semisolid state and from a semisolid to a solid state are define as the plastic limit (PL) and the shrinkage limit (SL), respectively. These limit a referred to as Atterberg limits (**figure 1.6**).

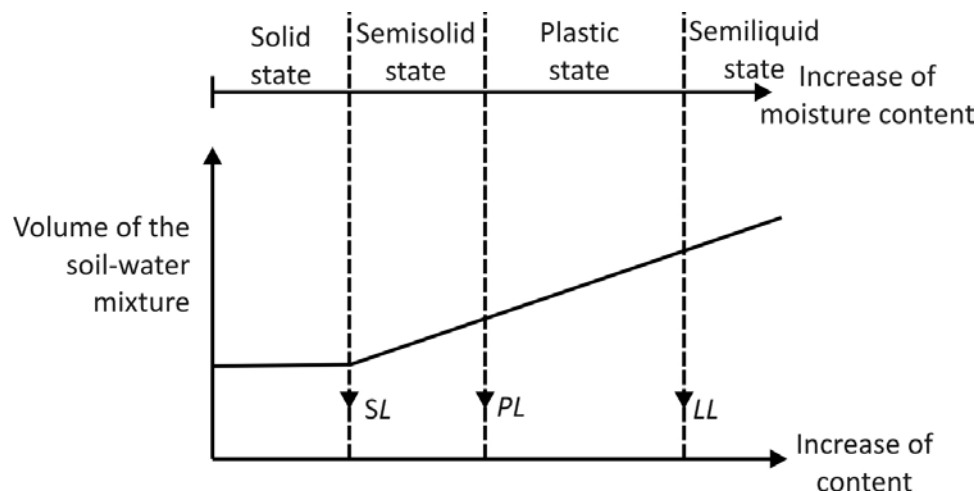


Figure 1.6 Definition of Atterberg limits

- The liquid limit of a soil is determined by Casagrande's liquid device (AST Testing Designation D-4318) and is defined as the moisture content at which a groove closure of $\frac{1}{2}$ in. (12.7 mm) occurs at 25 blows.

- The plastic limit is defined as the moisture content at which the soil crumble when rolled into a thread of $\frac{1}{8}$ in. (3.18 mm) in diameter (ASTM Testing Designation D-4318).
- The shrinkage limit is defined as the moisture content at which the does not undergo further volume change with loss of moisture (ASTM Testing Designation D-427). **Figure 1.6** shows this limit.

The difference between the liquid limit and the plastic limit of a soil is define as the plasticity index (PI), or

$$PI = LL - PL \quad [1.22]$$

Table 7 gives some representative values of liquid limit and plastic limit for several clay minerals and soils. However, Atterberg limits for various soils will variable considerable, depending on the soil's origin and the nature and amount of **clay** minerals in it.

Table 7 Typical Liquid and Plastic Limits for Some Clay Minerals and Soils

Description	Liquid limit	Plastic limit
Kaolinite	35-100	25-35
Illite	50-100	30-60
Montmorillonite	100-800	50-100
Boston Blue Clay	40	20
Chicago clay	60	20
Louisiana clay	75	25
London clay	66	27
Cambridge clay	39	21
Montana clay	52	18
Mississippi gumbo	95	32
Loessial soils in north and northwest China	25-35	15-20

